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Primary Author/Contact (Must be WSRC)	Location	Phone No.	Position
MIKE LEWIS	730-2B ~~704-2B~~/116	2-6847	E&CSD SGS GEOTECH. MGR

Organization Code	Organization (No Abbreviations)
E8300	SITE GEOTECHNICAL SERVICES

Other Authors
IGNACIO ARANGO, FARHANG OSTADAN, MICHAEL LEWIS

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Ms. W. F. Perrin, Technical Information Officer
U. S. Department of Energy
Savannah River Operations Office
Aiken, SC 29801

DOE Contact:
Brent Gutierrez
2-7115
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(Phone) (803) 725-2321
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Quantification of Seismic Liquefaction Risk (U)

Authored By

Ignacio Arango
Geotechnical Services
Bechtel Corporation
San Francisco, California

Farhang Ostadan
Geotechnical Services
Bechtel National, Inc.
San Francisco, California

Michael R. Lewis
Geotechnical Services
Bechtel Savannah River, Inc.
Savannah River Site
Aiken, South Carolina

Brent J. Gutierrez
Department of Energy
Savannah River Site
Aiken, South Carolina

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ABSTRACT

Explicit goals of acceptable risk for natural phenomena hazards (earthquake, extreme wind, and flood) have been established by the Department of Energy (DOE) 1994. Closely associated to the earthquake risk is the issue of seismically-induced liquefaction. Because deterministic methods currently available to answer the question to whether a site is liquefiable or not are incapable of providing a clue as to the likelihood or risk of liquefaction, the application of the criteria to a given facility requires that alternative evaluation techniques be formulated.

This paper describes the application to a nuclear facility of a newly developed probabilistic methodology which rigorously accounts for geotechnical and seismologic uncertainties. The results of the analyses are compared with the acceptable levels of risk presented by DOE. This comparison is used to emphasize the power of the methodology as a tool in the decision-making processes.

NOMENCLATURE

CSR	Cyclic stress ratio
CSRN	Normalized cyclic stress ratio
C_n	Factor to normalize standard penetration test N-Value for overburden pressure
C_q	Factor to normalize cone tip resistance
D_{bar}	Deaggregated earthquake distance
E	Earthquake
EPRI	Electric Power Research Institute
FS	Factor of safety against seismic liquefaction
g	Gravity
Hz	Hertz
LLNL	Lawrence Livermore National Laboratory
MSF	Magnitude scaling factor
M_{bar}	Deaggregated earthquake magnitude
N	SPT Blowcount
N_1	SPT Blowcount Corrected for "Overburden Pressure"
N_{60}	SPT Blowcount Corrected to 60% Energy
P	Compression Wave
PC	Performance Category
P_F	Permissible failure frequency limit
$P_E(L)$	Overall probability of liquefaction during earthquake E
P[L|E]	Conditional probability of liquefaction given E
P[E]	Probability that earthquake E occurs
P[L]	Overall Probability of Liquefaction
S	Shear Wave
SCPT	Seismic cone penetration test
SPT	Standard Penetration Test
$(N_1)_{60}$	Standard penetration test blow count
TR	Tobacco Road Formation
q_c	Cone tip resistance

$(q_c)_1$	Cone tip resistance normalized for overburden pressure
σ'_v	Vertical effective overburden pressure
τ_{MAX}	Maximum cyclic shear stress in soil
Σ	Sum over all earthquakes

INTRODUCTION

In a recent paper, Murray, et al, (1994) discuss DOE's explicit goals of acceptable risk for natural phenomena hazards, namely earthquakes, extreme winds, and floods. The acceptable risks were developed based on target probabilistic performance goals. These performance goals follow a graded approach beginning with normal-use facilities and ending with facilities involving hazardous or critical operations. Performance goals are defined by both qualitative expressions of acceptable behavior and target quantitative probabilities of exceedence/non-exceedence.

The DOE requirements for earthquake mitigation were established with the intent to provide an appropriate level of seismic protection for, a) occupant and public health and safety; b) the environment; c) production and research objectives; and d) potential property losses. A graded approach was implemented by defining five performance categories (PC-0 through PC-4), each with a performance goal. Performance category 0 covers items which require no seismic criteria. Performance category 4 (the most severe) is applicable to nuclear facilities comparable to commercial power reactors.

The quantitative performance goals are defined in terms of a permissible mean annual probability of unacceptable performance P_F (i.e., a permissible failure frequency limit). Seismically induced unacceptable performance should have an annual probability less than or approximately equal to these goals. Values of P_F for Performance Categories 1, 2, 3, and 4 are about 10^{-3}, about $5x10^{-4}$, about 10^{-4}, and about 10^{-5}, respectively. The qualitative descriptions of expected performance following design/evaluation levels of earthquake ground motions are shown in Table 1 for PC-1 through PC-4.

Earthquake ground shaking may cause the loss of strength of loose granular soils which results in settlement of buildings, landslides, failure of dams and embankments, and disruption of lifelines. This process of strength loss is called soil liquefaction. Numerous empirical relationships have been developed relating some soil characteristics which are indicative of liquefaction susceptibility and ground motions of varying intensities. Today, the relationship developed by Seed et al. in 1984 is perhaps the most widely used and accepted by the profession. This relationship, typical of the so-called deterministic models, provides a yes or no answer to the question whether liquefaction will or will not occur, and this answer is expressed numerically in terms of a factor of safety (FS).

An evaluation of the liquefaction potential for a critical facility located in Eastern U. S. was performed using deterministic qualitative and quantitative procedures. A generalized subsurface soil profile at this site is presented in Figure 1. In that evaluation, best estimate soil properties, along with the evaluation basis earthquake, were used in the analyses. Since the evaluation was based on deterministic methods, the effect of variability in soil properties and of earthquake motions on the liquefaction potential could not be integrated into the analysis. The purpose of the probabilistic evaluation presented here is to incorporate the effects of variation of the most significant soil properties and earthquake parameters in the evaluation of the annual liquefaction hazard at the site, and to compare this with the acceptable performance goals established by DOE, as referenced above. The evaluation is based on site-specific subsurface data, the Electric Power Research Institute (EPRI) seismic hazard study (EPRI 1986 and 1993) and a recently developed methodology for liquefaction hazard evaluation.

METHODOLOGY

The recently developed Advanced Seismic Hazard/Liquefaction Evaluation (ASHLE) method (Bechtel, 1991, Ostadan et al., 1991; Arango, I. and Ostadan, F., 1995) was used for the evaluation. In this method, the probability of liquefaction at a given site is obtained from

$$P_E(L) = P[L|E]\,P[E] \qquad (1)$$

where P_E (L) is the overall probability of liquefaction during the earthquake E; P[L|E] is the conditional probability of liquefaction given that earthquake E occurs; and P[E] is the probability that earthquake E occurs. The total overall probability of liquefaction is obtained by summing over all possible earthquakes:

$$P[L] = \sum_E P[L|E]\,P[E] \qquad (2)$$

The ASHLE methodology used models for conditional probability of liquefaction developed by Liao, et al. (1988). The models were developed based on statistical analyses (logit analysis followed by likelihood maximization) of a data catalog consisting of 278 observed cases of liquefaction/no liquefaction in Holocene deposits during forty earthquakes.

The Liao model, known as the local model for silty sands, was used as the basis for this evaluation. From this model, the probability of liquefaction can be obtained once the normalized blowcount data SPT $(N_1)_{60}$, and the normalized cyclic stress ratio (CSRN) are known. The normalized cyclic stress ratio is obtained from

$$CSRN = 0.65\,\frac{\tau_{MAX}}{\sigma'_v} \qquad (3)$$

where τ_{MAX} is the maximum seismic shear stress in the soil layer under consideration and σ'_v is the corresponding effective overburden pressure. The normalization of CSRN is performed so as to correct for the magnitude (duration) of the earthquake motion consistent with the approach used in the deterministic method (SEED, 1989).

In addition to the seismic models, the SPT blowcount corrected for overburden pressure and normalized to 60% energy, $(N_1)_{60}$, and the normalized cyclic stress ratio (CSRN) for each level of the earthquake motion (All Events E in Equation 2) are required as input to the ASHLE analysis.

<u>Seismic Models</u> - Current baselines for seismic hazards applicable to the site are described in investigations conducted by the EPRI (EPRI, 1986) and Lawrence Livermore National Laboratory (LLNL, 1989) and the revisions to the LLNL study.

Both the EPRI and the LLNL mean rock spectral hazard curves at 1.0 Hz and 2.5 Hz are shown in Figure 2. As shown, while the differences between the 1.0 Hz and the 2.5 Hz hazard curves are insignificant in each study, the LLNL results predict 3 to 4 times higher hazard compared to the EPRI results. The differences in these two results have been the subject of additional ongoing studies. In this paper only the liquefaction hazard evaluation, using the EPRI data for 2.5 Hz hazard curve are presented.

Use of the 2.5 Hz spectral hazard curve for liquefaction hazard evaluation was justified based on the results of several studies, such as Costantino (1994). In that study, a series of seismic free-field ground response analyses were performed using a total of 32 time histories (real and artificial). The effect of frequency content of the input motion on the cyclic stress ratios induced in the TR soils (i.e., the soils subject to the liquefaction evaluation) was thoroughly examined using both high pass and low pass band filters on the input motion. The study concluded that "the important characteristic of the input rock outcrop motion which controls the development of the induced shear stresses in the TR soils is the low frequency content of this motion as measured by the average spectral acceleration from 1 to 3 Hz."

A point on the spectral hazard curve, shown in Figure 2, defines the annual probability of the exceedence at a given ground motion level. This probability is calculated as the summation of the occurrences of ground motion exceedences over a range of source magnitudes (M_{bar}) and distances D_{bar}. To cover the wide range of ground motion intensities (from 1 cm/sec to 90 cm/sec in Figure 2) and the associated probabilities of exceedences, a total of ten points on the EPRI 2.5 Hz hazard curve was selected. For each point, through a hazard deaggregation process, the explicit contributions to the total hazard were obtained and displayed in space as earthquake magnitude and distance interval bins. For the above M_{bar} and D_{bar} pairs, rock acceleration response spectra were developed using region-appropriate seismic data. The resulting spectra were scaled so that the 2.5 Hz spectral amplitude matched that of the corresponding hazard curve point. For this, the computer program RASCAL, Silva, et al. (1987) was used. The program uses random vibration theory to calculate peak accelerations, velocity, and response spectra for specified earthquake source and propagation path parameters. Figure 3 presents the 10 spectra derived for the study. The spectra have been individually scaled so that the spectral values at 2.5 Hz maintain the appropriate spectral velocity, as defined by the EPRI mean 2.5 Hz spectral hazard curve. These spectra constitute the rock motion target spectra for which spectra-compatible time histories are generated for ground response analyses.

It may be noted that some of the target spectra in this figure are high. For example, the spectrum with the 2.5 Hz spectral velocity of 90 cm/s requires a spectral acceleration greater than 4.3g. However, the hazard level corresponding to these ground motions is about 3.5×10^{-7}, an extremely rare event.

Soil Characterization - For ground response analyses and liquefaction hazard evaluation, the soil properties consisting of the low-strain shear wave velocity, strain-dependent shear modulus and damping, and the insitu SPT blow-count data are required.

Low-strain shear wave velocity measurement from ten seismic piezo cone penetration probes (SCPTs) down to a depth of 170 feet below site ground surface, and two deep velocity (P and S) logged boreholes to a depth of about 1,000 feet below site ground surface were utilized.

A composite shear wave velocity for the ten SCPTs is shown in Figure 4. Table 2 presents the shear wave velocity, by layer, for these SCPTs, while Table 3 gives their respective layer thicknesses. Ultimately, the shear wave velocities presented in Table 2 and the layer thicknesses reported in Table 3 were used in a series of ground response analyses to represent the variability of shear wave velocity in the upper 170 feet of the subsurface profile.

To construct a complete shear wave velocity profile for ground response analyses, the deep shear wave velocity profiles were coupled with the ten shallow SCPTs. The deep and shallow profiles were randomly selected, paired and used in a Monte Carlo-type analysis.

For response analyses, site-specific modulus reduction and damping ratio vs. strain amplitude relationships were utilized.

SPT $(N_1)_{60}$ Values - The conditional probability of the liquefaction model requires the normalized SPT blow count, $(N_1)_{60}$, as a measure of the soil's resistance to liquefaction. Deterministic analyses show that the Formation in question (i.e., TR3/TR4 layer) has the lowest FS against liquefaction. Hence, the probabilistic liquefaction hazard analysis was restricted to SPT, N-values from this layer.

Available penetration resistance data for the site come from numerous boreholes with SPT, N results and SCPT probes. The borehole information consisted of two sets of data. The first set was from the original investigation performed for the facility (four underground storage tanks), while the second set was from the current study. The SCPT probes were all done as part of the current study. A rigorous quality assurance program was implemented for the current study. This included prior site-specific measurement of SPT hammer energy (measured as 60%). For the original investigations, however, documentation was not as complete. For example, the type of SPT hammer used was not recorded nor were SPT hammer energies measured. Thus, utilizing the previous SPT data required an evaluation.

The evaluation of the previous SPT data was accomplished by performing a statistical comparison of the previous uncorrected SPT data with the current uncorrected SPT data. The results showed that the previous uncorrected SPT N value data were, on the average, 1.6 times higher than the N value data of the current study. The difference suggests that the previous hammer energy was about 40%. Thus, all of the previous SPT N value data was divided by 1.6 to convert to an equivalent N_{60} value. Subsequently, all N_{60} data was converted to $(N_1)_{60}$ using the relation:

$$(N_1)_{60} = N_{60} \times C_n \qquad (4)$$

where C_n equals $(1 / \sigma'_v)^{0.5}$

The cone results SCPT data were also used to determine $(N_1)_{60}$. First, SCPT tip resistances (q_c) were corrected for overburden pressure using:

$$(q_{c1}) = C_q \times q_c \qquad (5)$$

where, $C_q \cong 1.63\ (\sigma'_v)^{-0.7}$, Seed, Idriss, and Arango (1983). Next, q_{c1} was converted to $(N_1)_{60}$ using the following relationship:

$$(N_1)_{60} = 0.285 q_{c1} - 2.1399 \qquad (6)$$

This was accomplished using site-specific statistical regression analysis developed for each subsurface layer using paired data from SCPT probes and SPT boreholes located no greater than 10 feet apart.

In addition to the site-specific SPT and SCPT data, geostatistical simulations were performed to develop additional representative data for the soils. Figure 5 shows the simulated $(N_1)_{60}$ data superimposed on the measured $(N_1)_{60}$. In general, the simulated values appear to agree with the measured data both in trend with depth and in the amount of scatter, suggesting that a reasonable simulation has been performed. The measured, simulated, and combined (measured plus simulated) $(N_1)_{60}$ values for the potentially liquefiable soil layer (i.e., TR3/TR4 layer) are presented in Table 4. All three sets of $(N_1)_{60}$ values (measured, simulated, and combined) were used in the ASHLE analysis to investigate the sensitivity of the final results to the simulated values.

Cyclic Shear Strength - The conditional probability of liquefaction models developed by Liao et al. (1988) are based on the statistical analysis of field data corresponding to Holocene deposits. For application to the study site, the model was modified to reflect the increased cyclic shear strength of the soils due to age, which also includes adjustments for overconsolidation and sample disturbance. Depending upon the degree of conservatism to be adopted for the analyses, the combined effect of the above adjustment factors had a numerical value of either 1.35 or 1.9. Both of these values were used in the analyses.

Cyclic Shear Strength Magnitude Scaling Factors - The magnitude scaling factors (MSF) developed by Seed and Idriss (1982) have been widely used. They are based on laboratory test data and statistical studies relating the number of significant strong motion cycles to earthquake magnitude. However, reliability of these factors has never been confirmed. Since publication of the MSF by Seed and Idriss in 1982, several other studies have been performed. These include energy concepts (Arango, 1994), statistical evaluation (Loertscher, 1994), and seismologic evaluation (Ambraseys, 1988). They all suggest that the Seed and Idriss MSF are very conservative (overestimating the damaging effect) for small to moderate earthquakes (magnitudes less than 7) and underestimating the damaging effects for large earthquakes (magnitudes greater than 8). A comparison of the various relationships is shown in Figure 6. As shown in this figure, the curve recommended by Arango is in close agreement with that recommended by Ambraseys, both of which are more conservative than Loertscher. For the probabilistic liquefaction hazard evaluation, the magnitude scaling factors derived by Arango were adopted.

Free-Field Seismic Ground Response Analysis - Free-field ground response analyses were performed using the computer program SHAKE, (Idriss et al., 1992). In all SHAKE runs, the input motion was prescribed as outcrop at a depth of 960 feet below ground surface, corresponding to the top of rock.

Response analyses would theoretically include ten soil columns 10 SCPT velocity profiles (for the upper layers), two shear wave velocity profiles (for the deep soil layers), and ten acceleration time histories, for a total of 200 separate computer runs. To reduce this number for each input motion a total of eight soil columns were randomly selected from the set of ten soil columns: half of these were combined with one of the deep shear wave velocity profiles and the other half were combined with the other deep shear wave velocity profile. With this random selection, the variation of soil properties were considered and the number of SHAKE runs was reduced to eighty. The results of the SHAKE analyses in terms of effective CSR (Defined in Equation 2) at two depths within the TR3 and TR4 layers (the critical layer for liquefaction) were obtained.

ASHLE ANALYSES AND RESULTS

Using the EPRI deaggregated hazard data corresponding to 2.5 Hz hazard curves, the normalized SPT, $(N_1)_{60}$ data (histogram, normal, and lognormal distributions), the CSRs (histograms, normal, and lognormal distributions), a series of ASHLE analyses was performed. The results are shown in Figure 7. As shown in this figure, the results are not sensitive to the use of either measured, simulated, or combined $(N_1)_{60}$ data. The results change, however, from $5x10^{-6}$ to approximately $2.5x10^{-6}$ once the shear strength factor changes from 1.35 to 1.9. The cluster of nine data points shown in each group of results in Figure 7 is due to the combination of various cases from three distributions of CSR and $(N_1)_{60}$ data (histogram, normal, and lognormal distributions) indicating that the liquefaction hazard is insensitive to statistical characterization of CSR and $(N_1)_{60}$ values.

It can also be seen with reference to the DOE criteria referred to in the Introduction section of this document that the calculated liquefaction hazard levels meet the target seismic performance goals recommended for Performance Category 4 (i.e., for nuclear facilities). Thus, the analyses show that for this site the ultimate acceptable degree of safety under natural hazards is controlled by factors other than exposure to seismic liquefaction.

CONCLUSION

The methodology developed demonstrates one of the benefits to using probabilistic analyses; that is the ability to compare the relative risk of various hazards or accident scenarios for critical facilities or components. The results can also be combined with other probabilistic studies to determine the overall risk to a facility from a seismic event, or any other postulated accident. Thus, more informed decisions regarding where evershrinking funds should be focused can be made.

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TABLE 1. QUALITATIVE SEISMIC PERFORMANCE GOALS

PERFORMANCE CATEGORY	OCCUPANCY SAFETY	CONCRETE BARRIER	METAL LINER	COMPONENT FUNCTIONALITY	VISIBLE DAMAGE
1	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation.	Confinement not required.	Confinement not required.	Component will remain anchored, but no assurance it will remain functional or easily repairable.	Building distortion will be limited but visible to the naked eye.
2	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation.	Concrete walls will remain standing but may be extensively cracked; they may not maintain pressure differential with normal HVAC. Cracks will still provide a tortuous path for material release. Don't expect largest cracks greater than 1/2 inch.	May not remain leak tight because of excessive distortion of structure.	Component will remain anchored and majority will remain functional after earthquake. Any damaged equipment will be easily repaired.	Building distortion will be limited but visible to the naked eye.
3	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation.	Concrete walls cracked; but small enough to maintain pressure differential with normal HVAC. Don't expect largest cracks greater than 1/8 inch.	Metal liner will remain leak tight.	Component anchored and functional.	Possibly visible local damage but permanent distortion will not be immediately apparent to the naked eye.
4	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation.	Concrete walls cracked; but small enough to maintain pressure differential with normal HVAC. Don't expect largest cracks greater than 1/8 inch.	Metal liner will remain leak tight.	Component anchored and functional.	Possibly visible local damage but permanent distortion will not be immediately apparent to the naked eye.

TABLE 2. SUMMARY OF SHALLOW DEPTH SHEAR WAVE VELOCITIES (fps)

SCPT	FILL	TR1	TR2	TR3/4	DB1/3	DB4	DB5	ST	GC
Depth Range (ft)	-	0-27	27-38.6	38.6-61.5	61.5-89.2	89.2-100.7	100.7-111.1	111.1-165.3	165.3-170
SCPT1	1129	1176	1499	976	795	757	867	824	1325
SCPT5	975	1461	1302	1304	1183	977	1000	1134	1325
SCPT7	1086	1196	1110	1268	1155	1055	-	1041	1325
SCPT11	-	1085	1119	1222	1206	1092	1457	1780	1326
SCPT14	-	1086	982	1363	1076	682	-	1409	1325
SCPT18	1175	1379	1122	1308	1361	1236	1152	1178	1325
SCPT25	-	1269	1132	1132	831	747	-	1223	1325
SCPT26	1024	1331	1126	1167	951	909	800	2278	1325
SCPT29	976	1129	1112	1042	1365	1166	-	1469	1325
SCPT30	1034	1167	922	1080	1137	1087	888	953	1325

Note: SCPT - Seismic Piezocone Penetration Probe

TABLE 3. SUMMARY OF LAYER THICKNESS (ft)

SCPT	FILL	TR1	TR2	TR3/4	DB1/3	DB4	DB5
SCPT1	41	15	27	24	32	9	8
SCPT5	41	25	15	22	36	7	10
SCPT7	41	30	20	24	31	24	-
SCPT11	42	30	18	22	32	8	15
SCPT14	6	28	26	24	35	17	-
SCPT18	10	23	31	27	28	5	15
SCPT25	-	33	25	21	25	9	-
SCPT26	36	11	21	23	33	15	16
SCPT29	44	35	16	24	35	4	-
SCPT30	41	34	12	22	31	6	21

Note: SCPT - Seismic Piezocone Penetration Probe

TABLE 4. SUMMARY OF SPT N-VALUE DATA

Data Points	Location	$(N_1)_{60}$ Values
Measured (Total=50)	Boring SPT-14	9.8, 10.4, 6.7, 7.8, 7.7, 7.7, 7.0, 7.0, 8.1, 14.3, 13.6, 12.4, 11.2, 10.0
	Boring SPT-18	9.5, 17.1, 10.5, 9.3, 8.1, 8.6, 9.1
	Boring SPT-20	15.5, 8.4, 9.2, 9.8, 8.9, 8.7, 6.3, 6.9, 6.1, 8.3, 7.5, 9.5, 9.5, 13.6
	Boring SPT-23.3	9.0, 5.6, 4.8, 3.1, 4.6, 3.8, 4.5, 2.2, 2.9, 3.6, 3.6, 3.5, 4.9, 3.5, 4.8
Simulated (Total=21)	Tanks 1 - 4	5.4, 6.6, 5.2, 8.8, 11.4, 11.2
		12.0, 15.1, 8.3, 2.9, 5.3, 7.6, 10.0
		16.9, 3.4, 1.5, 4.2, 4.1, 6.9
		5.5, 6.9
Combined (Total=71)	Borings SPT - 14, 18, 20 & 23.3 and Tanks 1-4	9.8, 10.4, 6.7, 7.8, 7.7, 7.7, 7.0, 7.0, 8.1, 14.3, 13.6, 12.4, 11.2, 10.0
		9.5, 17.1, 10.5, 9.3, 8.1, 8.6, 9.1
		15.5, 8.4, 9.2, 9.8, 8.9, 8.7, 6.3, 6.9, 6.1, 8.3, 7.5, 9.5, 9.5, 13.6
		9.0, 5.6, 4.8, 3.1, 4.6, 3.8, 4.5, 2.2, 2.9, 3.6, 3.6, 3.5, 4.9, 3.5, 4.8
		5.4, 6.6, 5.2, 8.8, 11.4, 11.2
		12.0, 15.1, 8.3, 2.9, 5.3, 7.6, 10.0
		16.9, 3.4, 1.5, 4.2, 4.1, 6.9
		5.5, 6.9

FIGURE 1. GENERALIZED SUBSURFACE SOIL PROFILE

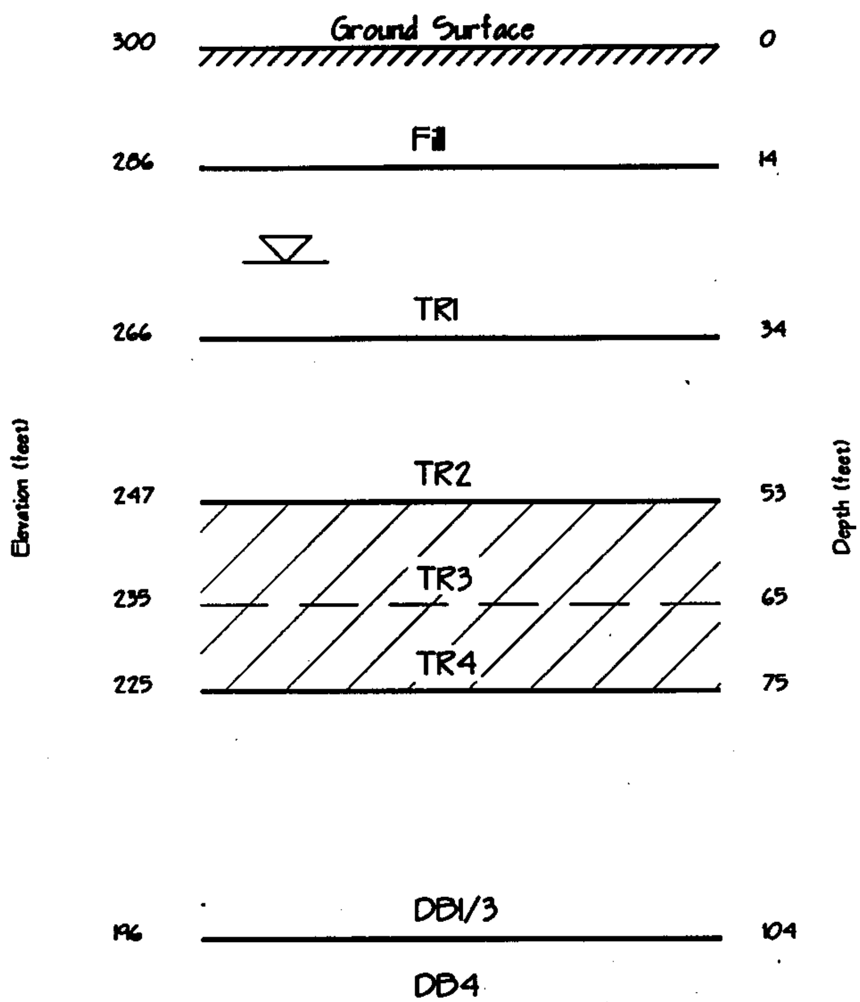


FIGURE 2. EPRI AND LLNL SPECTRAL HAZARD CURVES

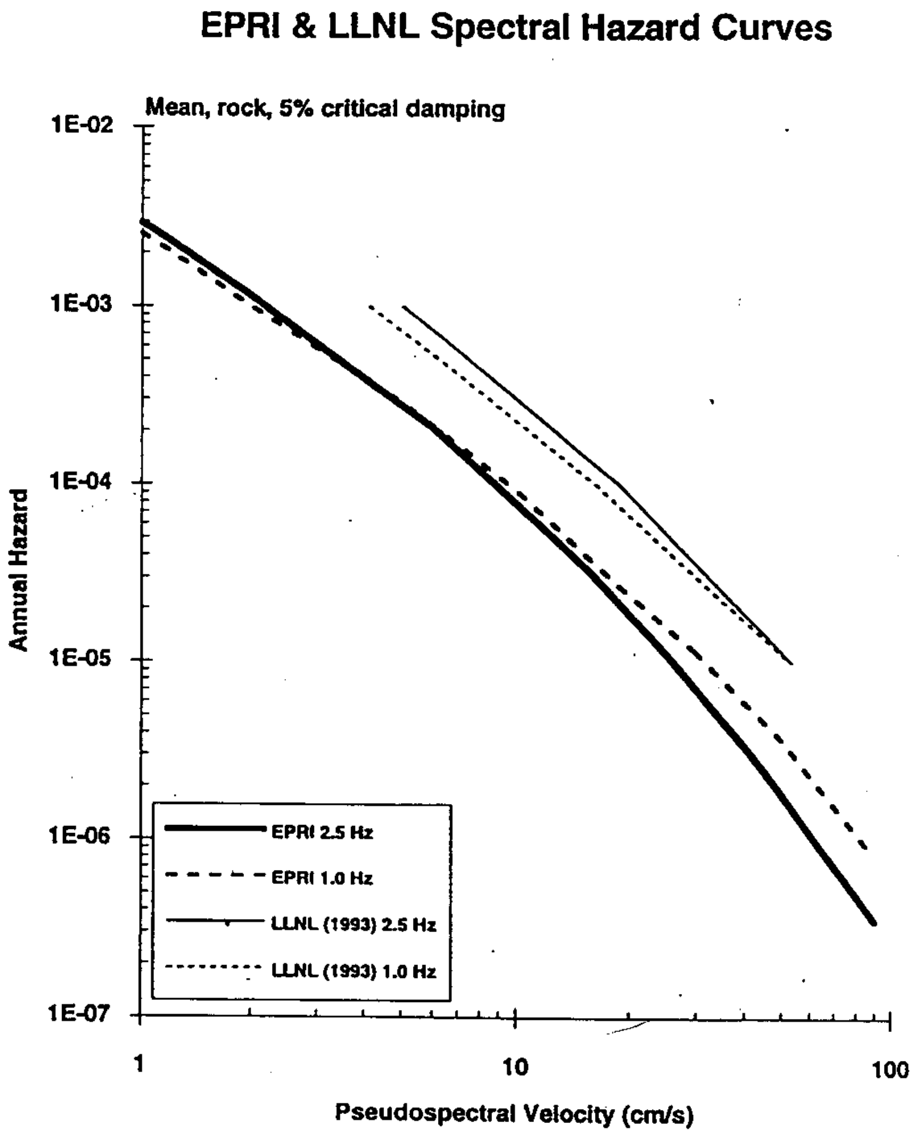


FIGURE 3. ACCELERATION RESPONSE SPRECTRA (5% DAMPING) DEVELOPED FOR THE ROCK MOTION

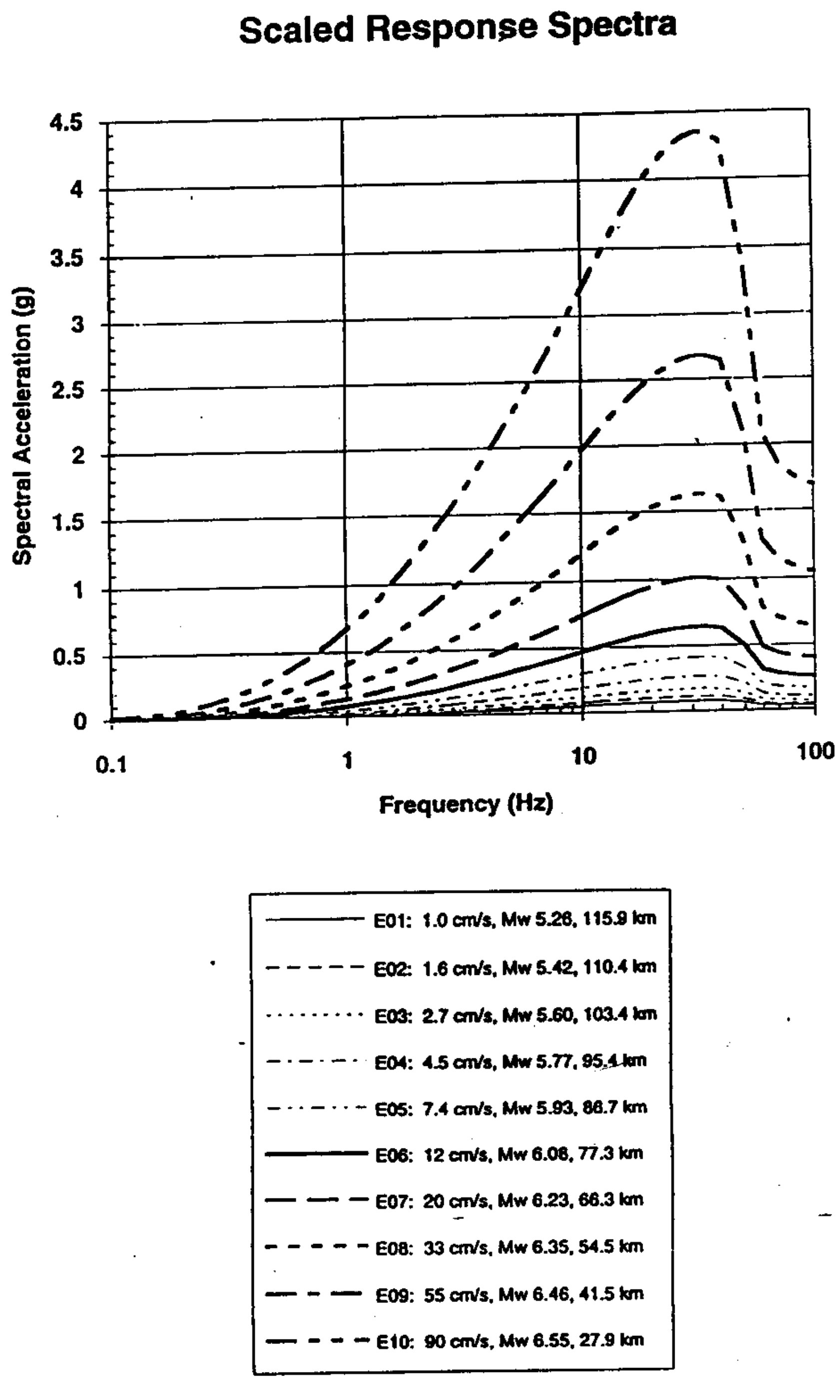


FIGURE 4. SHEAR WAVE VELOCITY PROFILES FROM SEISMIC CONE PROBES

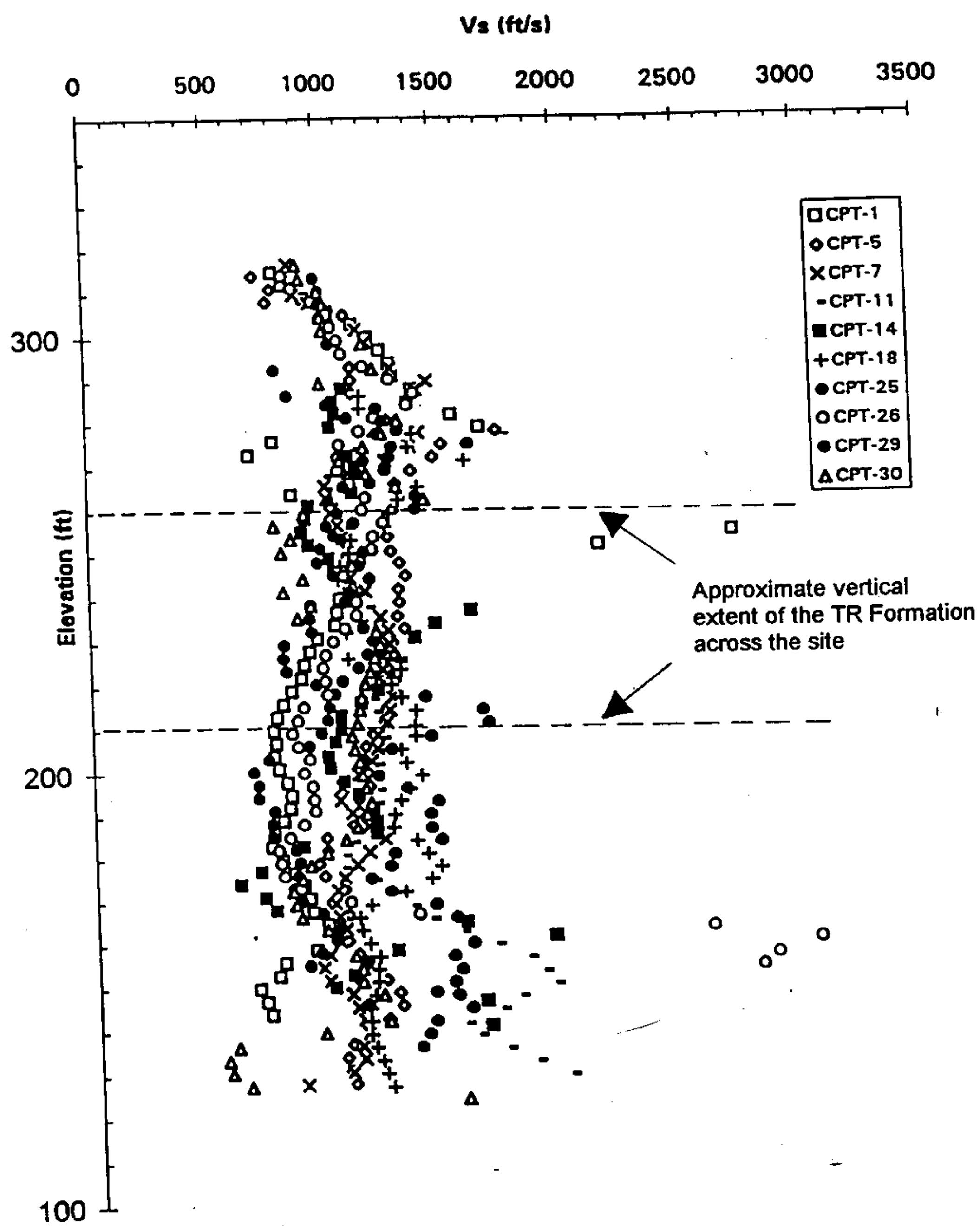


FIGURE 5. MEASURED AND SIMULATED $(N_1)_{60}$ VALUES

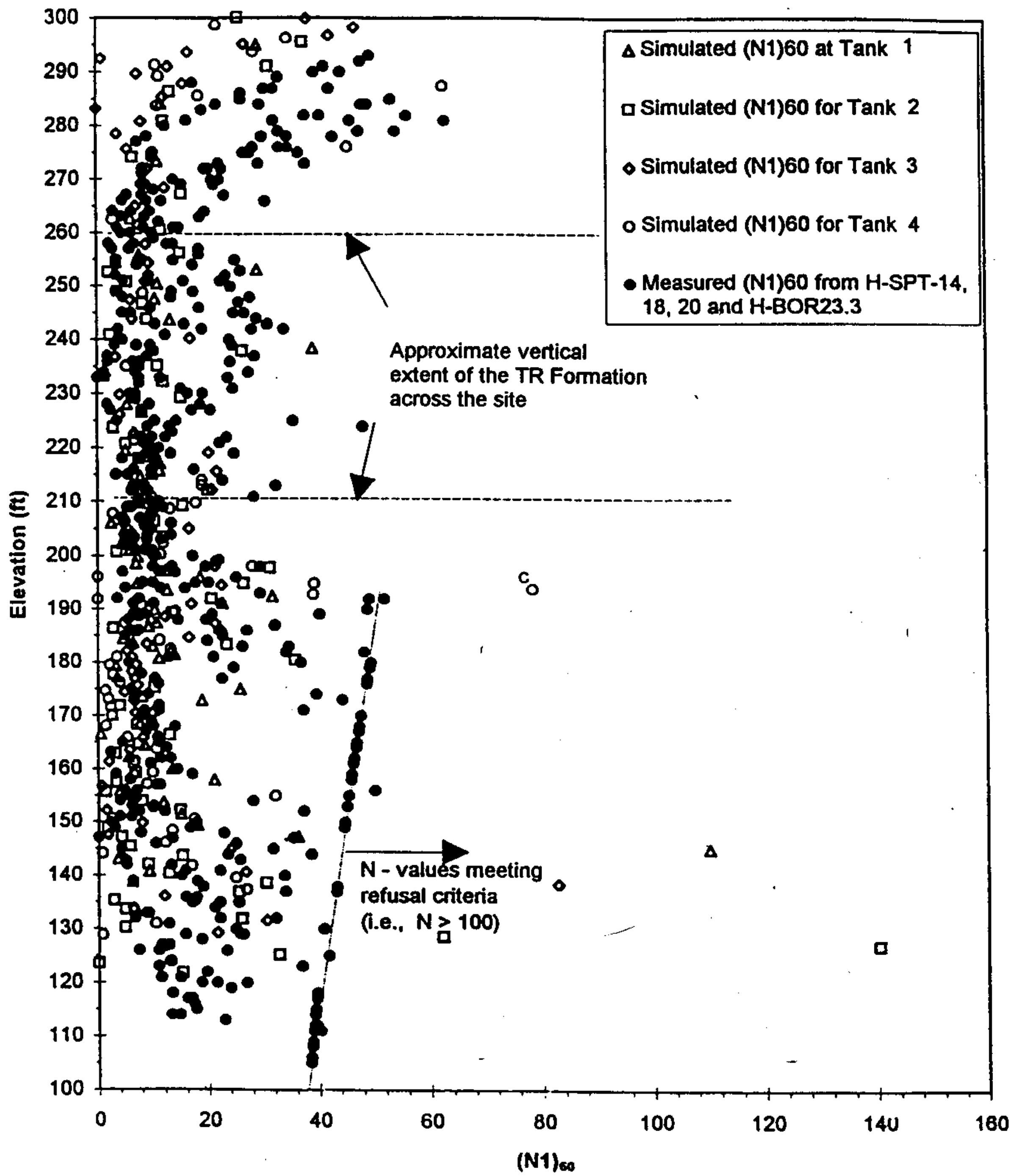


FIGURE 6. COMPARISON OF EARTHQUAKE MAGNITUDE SCALING FACTORS FROM VARIOUS SOURCES

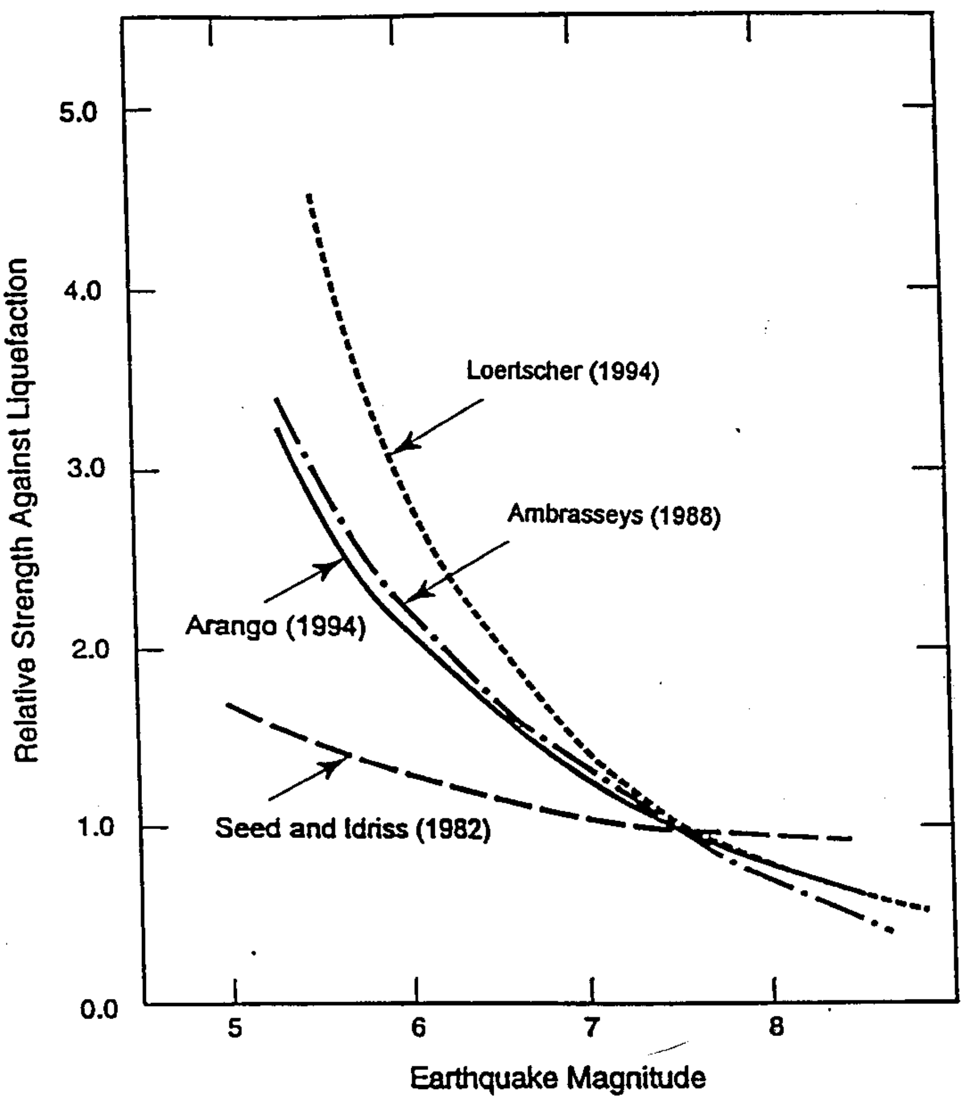
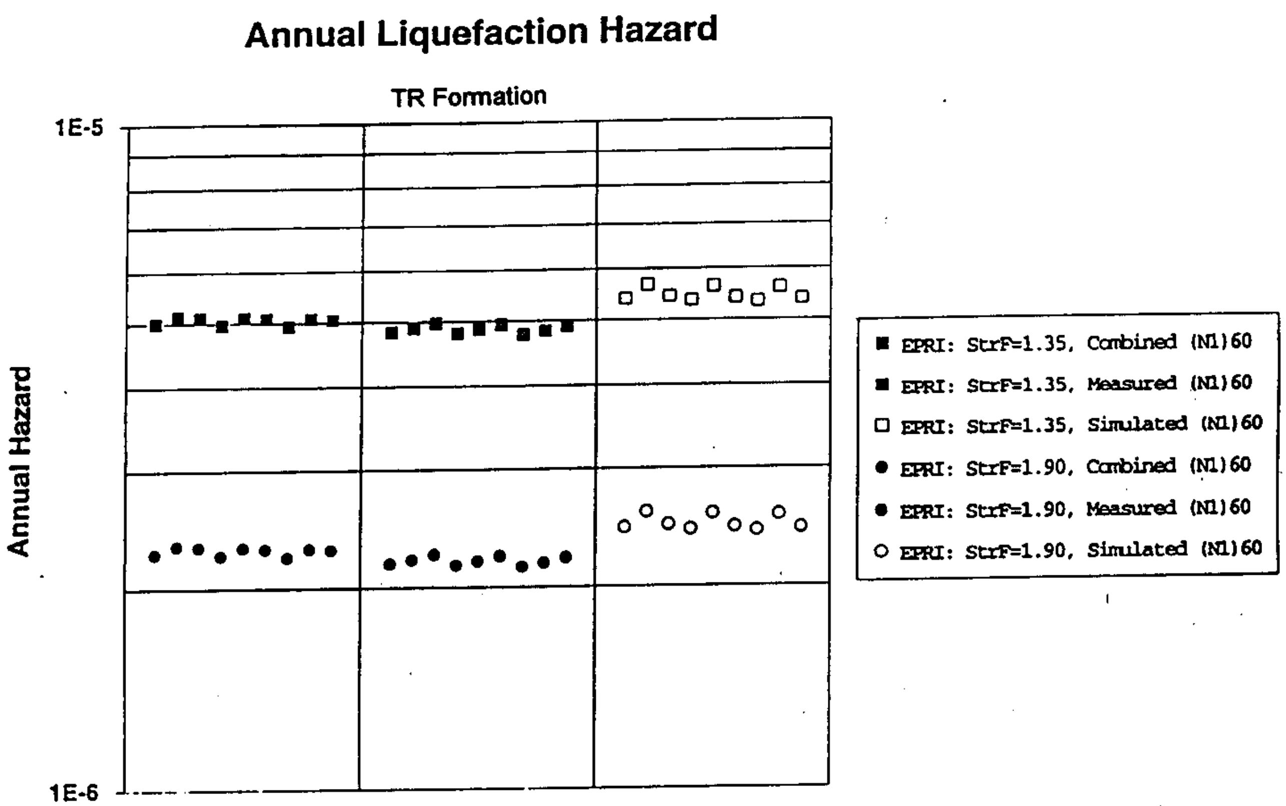


FIGURE 7. ANNUAL LIQUEFACTION HAZARD



Clusters of 9 points from combinations of the 3 (N1)60 distributions and the 3 CSR distributions